DRAFT REPORT

IN-DELTA STORAGE PROGRAM FLOODING ANALYSIS

Prepared for
Department of Water Resources
901 P Street
Sacramento, CA 94236

June 2003

URS Corporation
500 12th Street, Suite 200
Oakland, CA 94607

26814104

TABLE OF CONTENTS

Tables

Figures

Appendices

1.1 PURPOSE

The Department of Water Resources (DWR) is conducting feasibility-level engineering and environmental studies under the Integrated Storage Investigations Program. As part of the project evaluations, DWR is evaluating the technical feasibility and conducting engineering investigations for the In-Delta Storage Program. The engineering investigation will aim at developing solutions to enhance project reliability through improved embankment design and consolidation of inlet and outlet structures.

As part of this feasibility study, DWR requested that URS Corporation (URS) undertake a detailed risk analysis and integrate the physical design with a desirable level of protection against seismic, flooding, operational, environmental and economic risks.

1.2 SCOPE OF WORK

This report presents the evaluation of flood risks associated with wind generated wave runup and hypothetical breach failures of the re-engineered In-Delta Storage Reservoir islands at Webb Tract and Bacon Island. The specific tasks proposed under the statement of work for Task Order No. IDS-0702-1747-003 are presented below:

Task 1: Collect and Review Existing Information

Review historical flood events in the Delta from CALFED and previous studies, including information being collected and compiled by the DWR Floodplain Management program and the Corps of Engineers Comprehensive Study. Review studies of predicted flood events in the Delta and their associated probabilities, including preliminary and final computer model runs and input and output data being completed by DWR and Corps of Engineers. Obtain hydraulic data for the 50-, 100-, and 300-year flood events in the Delta. Review and compile historic data from relevant gauging stations within the Delta.

Task 2: Perform Wave Run-up and Wind Set-up Analysis

Perform analysis around the islands to estimate maximum fetches at different levee sections and estimate wave runup and wind setup values for the 50-, 100- and 300-year flood events. Estimate the probability of overtopping based on the proposed re-engineered project. Provide recommendations for adequate crest elevations for the project.

Task 3: Perform Embankment Breach Analysis

Perform a dam breach analysis to estimate rate of flow releases for both inward and outward embankment failures occurring during an operational event. These analyses will use the potential failure scenarios developed in the operational demand tasks. The objective of this activity is to provide sufficient input to estimate the impacted areas and to quantify the consequences of failure from an uncontrolled release. This task will consider various water levels scenarios between the proposed reservoir islands and the Delta sloughs. The hypothetical breach scenarios will be centered at different locations around the reservoir islands to cover a reasonable range of flooding and impacted areas.

2.1 DATA REVIEW

Historical data including flood and tide elevations in the Delta region were obtained from previous studies conducted by CALFED, DWR, U.S. Army corps of Engineers (USACE) and URS. Flood stage data are published in the reports titled *"Sacramento-San Joaquin Delta Levee Rehabilitation Study"* by CALFED (September 1998) and "Sacramento-San Joaquin Delta, California, Special Study, Hydrology" by USACE (February 1992). CALFED reported only the 100-year flood stage values for the Delta region, whereas USACE reported 50-, 100-, and 300-year flood stage values for the region. These design flood stage data are presented in Table 2-1.

Table 2-1
Flood Stage Data Estimated by CALFED and USACE

Reservoir Island	Average Design Flood Stage (feet – NGVD 1929)			
	50-year	100-year		300-year
	USACE	USACE	CALFED(1)	USACE
Webb Tract	6.8	7.0	6.9	7.2
Bacon Island	6.9	7.2	7.2	7.5

(1) Average flood stage. For specific stage elevation around each island, see Appendix A.

Wind generated wave runup values estimated by CALFED (1998) were obtained and reviewed for the present study. The report titled "In-Delta Storage Program Risk Analysis" by URS (November 2001) evaluated flood overtopping risks due to wind-generated wave action based on wave runup estimates made by CALFED (1998) and flood stage data provided by USACE (1992) for the existing levee geometric conditions.

2.2 ANALYSIS PARAMETERS

2.2.1 Embankment Geometry

The embankment crest of the proposed reservoir islands at Webb Tract and Bacon Island will be constructed to at least elevation +10.0 feet. For embankment sections adjacent to Franks Tract and Mildred Island, the following geometric shapes were used for the slough side of the embankment:

- At Franks Tract and Mildred Island: A bank slope of 3:1 (H:V) with no berms (rock-berm option)
- At Franks Tract: A composite bank slope with a horizontal berm at elevation + 2.0 feet, the slope below the berm as 2.14:1 (H:V) and above the berm as 3:1 (H:V)
- At Franks Tract: A composite bank slope with a horizontal berm at elevation + 6.0 feet, the slope below the berm as 2.14:1 (H:V) and above the berm as 3:1 (H:V)

- At Mildred Island: A composite bank slope with a horizontal berm at elevation + 3.0 feet, the slope below and above the berm as 3:1 (H:V).

For slough side slopes of embankment sections that are not adjacent to Franks Tract and Mildred Island, two geometric options were considered as follows:

- Rock-berm option with a bank slope of 3:1 (H:V) with no berms.
- Bench option with varying bench elevations and widths such that average slope ranges from approximately 3:1 (H:V) to 5:1 (H:V)

The embankment slope on the reservoir side is designed as 3:1 (H:V) above the maximum water surface elevation (WS EL) of +4.0 and 10:1 (H:V) below.

2.2.2 Wind Wave Runup

Wind-wave runup estimates for this study are analyzed for different embankment geometric configurations and fetch lengths. For embankment sections adjacent to Franks Tract and Mildred Island, wind wave runup values are estimated using the geometric shapes described in Section 2.2.1 with riprap armor in place. For embankment sections that are not adjacent to Franks Tract and Mildred Island, wind wave runup values are estimated using an average slope of 3:1 (H:V) with riprap armor in place on the slough side of the embankment.

The remnant levees and marsh areas in Franks Tract were not considered in the wind wave run-up analysis. The full fetch across Franks Tract was used to calculate the wave runup on the slough side of the Webb Tract embankment.

2.3 ANALYSIS CRITERIA

2.3.1 Reservoir Stages and Slough Water Levels

Minimum and maximum storage water levels in Webb Tract and Bacon Island and minimum, average and average-high tide levels in the surrounding sloughs that were used in the analyses are presented in Table 2-2. Minimum and maximum reservoir island water levels were obtained from the URS report (2001). Minimum, average and average-high tide levels in the sloughs were obtained from the USACE report (1992).

Table 2-2
Reservoir Stages and Slough Water Levels

Reservoir Island	**Webb Tract**	**Bacon Island**
Minimum WSEL in Reservoir	-8.0 (1)	-8.0 (1)
Maximum WSEL in Reservoir	+4.0	+4.0
Minimum Tide Level in Slough	-1.0	-1.0
Average Tide Level in Slough	+1.5	+1.5
Average-High Tide Level in Slough	+3.5	+3.5

(1) This condition exists for about five months per year during the periods of emptying and filling of the reservoir (URS, 2001).

2.3.2 Breach Evaluation Criteria

Three geometric configurations were evaluated in the breach analyses to determine potential impacts on adjacent levees. Both reservoir islands are surrounded by sloughs of varying widths and depths. For the breach analysis, the sloughs surrounding Webb Tract and Bacon Island are categorized into three groups: narrow, medium, and wide. Slough widths and bottom elevations used in the analysis are presented in Table 2-3.

The elevations of the bottom of the slough vary around the islands. Based on survey data, the slough bottom elevation ranges from about –18 to –50.0 feet and averages about -25.0 feet. To provide conservative peak velocity estimates, the slough bottom elevations were set at –25.0 feet for the three typical channel widths (see Table 2-3). The elevations of the bottom of the proposed reservoir were taken from the DWR area-capacity curves (DWR, 2001).

Table 2-3
Typical Embankment Geometry for Breach Analysis

Reservoir Island	Crest Elevation	Slough Bottom Elevations			New Res. Bottom Elevation
		Narrow (400 ft)	Medium (1,000 ft)	Wide (3,000 ft)	
Webb Tract	+10.0	-25.0	-25.0	-25.0	-20.0
Bacon Island	+10.0	-25.0	-25.0	N/A	-18.0

To evaluate the impacts of the embankment breach scenarios on adjacent levees, a velocity range of 8 feet per second (fps) to 10 fps was used as the threshold for failure of adjacent levees. These threshold velocities apply to slough-side slopes covered by riprap (D_{50} of about 1 foot) (Neill, 1973). For velocities less than this range, it was assumed that the adjacent levees would not fail.

To evaluate the impacts of overtopping of adjacent levees during a hypothetical reservoir breach, it is assumed that the crest elevations of adjacent levees are 8.0 feet.

2.3.3 Freeboard Criteria

The embankment crest elevations shall be the larger of the following two criteria (Calfed, 2002):

1) The maximum reservoir water storage elevation (+4 feet MSL) plus the wind wave runup plus setup on the reservoir. If wind wave runup plus setup is less than 3 feet, then a freeboard of 3 feet should instead be added to the maximum water storage elevation, or

2) The water surface elevation of the design flood event on the river side plus the wind wave runup plus setup. If the wind wave runup plus setup is less than 3 feet, then a freeboard of 3 feet should instead be added to the water surface elevation of the design flood event.

3.1 METHODOLOGY

Freeboard requirements at Webb Tract and Bacon Island reservoirs were evaluated based on design flood stages and wind wave characteristics estimated for the Sacramento-San Joaquin Delta region. Embankment crest elevations of the reservoir islands were designed to protect the embankments from overtopping due to extreme flooding and wind loading conditions on the surrounding water bodies. Flood stage data used to design crest elevations of the reservoir islands are described in Section 2.1. Design wind wave characteristics and typical embankment geometric configurations used to calculate wave runup and setup values for the reservoir islands are described in Section 3.2.

3.2 WIND WAVE ANALYSIS

A wave runup analyses for sloughs surrounding Webb Tract and Bacon Island were performed to estimate freeboard requirements for the reservoir embankments. Wave runup (R) is defined as the vertical height above still-water level (SWL) to which water from an incident wave will run up the face of a structure. The wave runup analyses involved estimating wave characteristics such as wave height (H_{mo}) and wave period (T_m) from wind velocities (U_f) and reservoir fetch length (F). The analyses indicated that wind waves at Webb Tract (adjacent to Franks Tract) and Bacon Island (adjacent to Mildred Island) are fetch-limited deep water waves (Shore Protection Manual by USACE, 1984).

3.2.1 Effective Fetch Length

Webb Tract and Bacon Island reservoirs are surrounded by water bodies of varying fetch lengths. These fetch lengths were categorized into three typical lengths: short, medium, and long. Table 3-1 provides the approximate station locations of typical short, medium, and long fetch length categories and the stations adjacent to Franks Tract and Mildred Island. Figures 3-1 and 3-2 show the station locations (described in Table 3-1) for the Webb Tract and Bacon Island reservoirs, respectively.

Table 3-1
Embankment Station Locations at Webb Tract and Bacon Island Reservoirs

Reservoir Island	Levee Station (feet)			
	Adjacent to Franks Tract and Mildred Island	Slough Section with "Short" Fetch Length	Slough Section with "Medium" Fetch Length	Slough Section with "Long" Fetch Length
Webb Tract	70+00 to 220+00(1)	590+00 to 680+00	0+00 to 70+00 220+00 to 290+00	290+00 to 350+00 350+00 to 590+00
Bacon Island	60+00 to 200+00(2)	200+00 to 250+00 620+00 to 700+00	0+00 to 60+00 250+00 to 350+00 350+00 to 570+00 570+00 to 620+00 700+00 to 750+00	N/A

(1) Section adjacent to Franks Tract
(2) Section adjacent to Mildred Island

The maximum effective fetch length for each category, in addition to the sections adjacent to Franks Tract and Mildred Island, were measured and used in the analysis (see Table 3-2). These effective fetch lengths were calculated using procedures given in the Shore Protection Manual (USACE, 1984) and ACER Technical Memorandum No 2 (USBR, revised 1992).

Table 3-2
Effective Fetch Length at Webb Tract and Bacon Island Reservoirs

Reservoir Island	Effective Fetch Length (miles)			
	Adjacent to Large Water bodies	Slough Section with "Short" Fetch Length	Slough Section with "Medium" Fetch Length	Slough Section with "Long" Fetch Length
Webb Tract	3.22 (1)	0.34	0.60	1.29
Bacon Island	2.04 (2)	0.39	0.69	N/A

(1) Effective fetch length adjacent to Franks Tract
(2) Effective fetch length adjacent to Mildred Island

3.2.2 Design Wind Velocity

Wind velocities for the "fastest mile of record" were obtained from generalized charts published by USACE (1976) and USBR (1981). The "fastest mile of record" was used to calculate average wind velocities associated with the minimum wind duration required to generate the reservoir wind wave spectrum. The estimated fastest mile of record wind velocities at the reservoir sites for winter, spring, summer and fall are 60, 56, 40, and 60 miles per hour, respectively. These values are wind velocities over land at elevation 25 feet. The generalized charts for the fastest mile of record published by USACE (1976) are included in Appendix B.

3.2.3 Wind Wave Runup

The estimate of wave runup requires both wind wave and reservoir embankment characteristics. These characteristics are (1) minimum wind duration (t_d) to generate the wind wave spectrum, (2) average wind velocity over water (U_w), (3) wind stress factor (U_A), (4) significant wave height (H_{mo}), (5) wave period (T_m), and (6) slope and roughness characteristics of the embankment face.

To calculate wave runup values at embankment sections adjacent to Franks Tract and Mildred Island, which are both flooded, the geometric shapes described in Section 2.2.1 were used for the slough side of the embankment with riprap armor in place. For wave runup calculations at embankment sections that are not adjacent to Franks Tract and Mildred Island, an average slope of 3:1 (H:V) was used with three typical fetch lengths (short, medium, and long) for the slough side of the embankment with riprap armor in place.

Equations and design charts used to estimate wave runup values are presented in the Shore Protection Manual (USACE, 1984) and are included in Appendix B.

3.2.4 Wind Setup

Estimates of wind setup (S) resulting from winds on the slough side of the Webb Tract and Bacon Island reservoirs were also made. Wind setup is a general tilting of water surface due to shear stress caused by winds. Wind setup was estimated using the procedure published by USBR (1981). The wind setup estimates require (1) average wind velocity over water (U_w), (2) effective slough side fetch length (F), and (3) average water depth at slough side (d_w).

3.3 RESULTS

Wave action from wind, calculated by adding wave runup (R) and wind setup (S), is used to evaluate the freeboard requirements at Webb Tract and Bacon Island reservoirs. Tables 3-3 and 3-4 present the 50-, 100-, and 300-year design flood stages (USACE, 1992), estimated wave runup plus setup values, and the resulting maximum flood elevations during 50-, 100-, and 300-year flood events at Webb Tract and Bacon Island reservoirs, respectively. As mentioned earlier, the freeboard requirement for the project is 3 feet on the 100-year flood stage or maximum wind wave runup plus setup, whichever is greater. The results indicate that the maximum wind wave runup plus setup is 1.8 feet for Webb Tract and 1.4 feet for Bacon Island; therefore, the freeboard required for the embankments around both Webb Tract and Bacon Island is 3 feet on the design flood event. The embankments would need to have crest elevations of +10.1 feet at Webb Tract and +10.3 feet at Bacon Island to have sufficient freeboard. This provides an additional freeboard above the maximum 100-year flood elevation ranging from 1.3 to 2.5 feet at Webb Tract and from 1.7 to 2.5 feet at Bacon Island. Tables 3-3 and 3-4 show that the crest elevations are also sufficient to prevent overtopping due to the 300-year flood event.

Table 3-3
Estimated Wind Wave Runup and Reservoir Setup at Webb Tract Reservoir

Webb Tract Embankment Station	Wind Wave Runup + Setup (feet)	Design Flood Stage (USCAE, 1992) (feet)			Maximum Flood Elevation (= wind wave runup + setup + design flood stage) (feet)			Section used to Estimate Wave Runup *(See Tables 3.1 and 3.2)*
		50-year	100-year	300-year	50-year	100-year	300-year	
0+00 to 70+00	0.8 (1)	6.8	7.0	7.2	7.6	7.8	8.0	Medium
220+00 to 290+00	0.8 (1)	6.8	7.1	7.2	7.6	7.9	8.0	Medium
290+00 to 350+00	1.1 (1)	6.8	7.0	7.2	7.9	8.1	8.3	Long
350+00 to 590+00	1.1 (1)	6.8	7.0	7.2	7.9	8.1	8.3	Long
590+00 to 680+00	0.6 (1)	6.8	7.0	7.2	7.4	7.6	7.8	Short
70+00 to 220+00	1.8 (2) 1.8 (3) 0.6 (4)	6.8	7.0	7.2	8.6	8.8 8.8 7.6	9.0	Adjacent to Franks Tract

(1) For average bank slope of 3:1 (H:V) with quarrystone riprap (see Appendix B).
(2) For average bank slope of 3:1 (H:V) with quarrystone riprap (see Appendix B).
(3) A composite bank slope with a horizontal berm at + 2.0 feet and with quarrystone riprap.
(4) A composite bank slope with a horizontal berm at + 6.0 feet and with quarrystone riprap.

Table 3-4
Estimated Wind Wave Runup and Reservoir Setup at Bacon Island Reservoir

Bacon Island Embankment Station	Wind Wave Runup + Setup (feet)	Design Flood Stage (USCAE, 1992) (feet)			Maximum Flood Elevation (= wind wave runup + setup + design flood stage) (feet)			Section used to Estimate Wave Runup *(See Tables 3.1 and 3.2)*
		50-year	100-year	300-year	50-year	100-year	300-year	
0+00 to 60+00	0.8 (1)	6.9	7.3	7.5	7.7	8.1	8.3	Medium
200+00 to 250+00	0.6 (1)	6.9	7.2	7.5	7.5	7.8	8.1	Short
250+00 to 350+00	0.8 (1)	6.9	7.1	7.5	7.7	7.9	8.3	Medium
350+00 to 570+00	0.8 (1)	6.9	7.2	7.5	7.7	8.0	8.3	Medium
570+00 to 620+00	0.8 (1)	6.9	7.3	7.5	7.7	8.1	8.3	Medium
620+00 to 700+00	0.6 (1)	6.9	7.3	7.5	7.5	7.9	8.1	Short
700+00 to 750+00	0.8 (1)	6.9	7.3	7.5	7.7	8.1	8.3	Medium
60+00 to 200+00	1.4 (2) 1.4 (3)	6.9	7.2	7.5	8.3	8.6 8.6	8.9	Adjacent to Mildred Island

(1) For average bank slope of 3:1 (H:V) with riprap (see Appendix B).
(2) For average bank slope of 3:1 (H:V) with quarrystone riprap (see Appendix B).
(3) A composite bank slope with a horizontal berm at +3.0 feet and with quarrystone riprap.

The estimates of probability of embankment failure under varying loading scenarios are presented in the URS Risk Analysis report. The reader is referred to that report, which combines the probabilities of embankment failures under seismic, flood, and operational conditions and estimates the aggregated risk of failure including the associated consequences of failure to the existing resources within the project area.

3.4 RESERVOIR-SIDE WAVE RUNUP AND SETUP ANALYSIS

Wave runup and setup were calculated for the reservoir sides of Webb Tract and Bacon Island to check the adequacy of the embankment freeboard due to wave action within the reservoirs. To estimate the wave runup and setup for the reservoir sides of Webb Tract and Bacon Island, the following design parameters were used:

- Fetch lengths of 3.68 and 4.06 miles for Webb Tract and Bacon Island, respectively, were calculated using the procedures given in the Shore Protection Manual (USACE, 1984) and ACER Technical Memorandum No 2 (USBR, Revised 1992).
- As for the slough-side analyses, fastest mile of record wind speed of 60 miles per hour was obtained from the Generalized Charts for the Fastest Mile of Record published by USACE (1976) and ACER Technical Memorandum No 2 (USBR, revised 1992).
- Reservoir side embankment slope of 3H:1V above elevation +4.0 feet, with riprap armor assumed to be in place for both reservoir islands.

Based on the above design conditions, the wave runup plus setup values on the reservoir sides were estimated to be 2.0 feet and 2.2 feet for Webb Tract and Bacon Island, respectively. Therefore, with maximum reservoir water storage elevation at elevation +4.0 feet, both reservoir islands would have sufficient freeboard with crest elevations at 10.1 and 10.3 feet for Webb Tract and Bacon Island, respectively.

URS

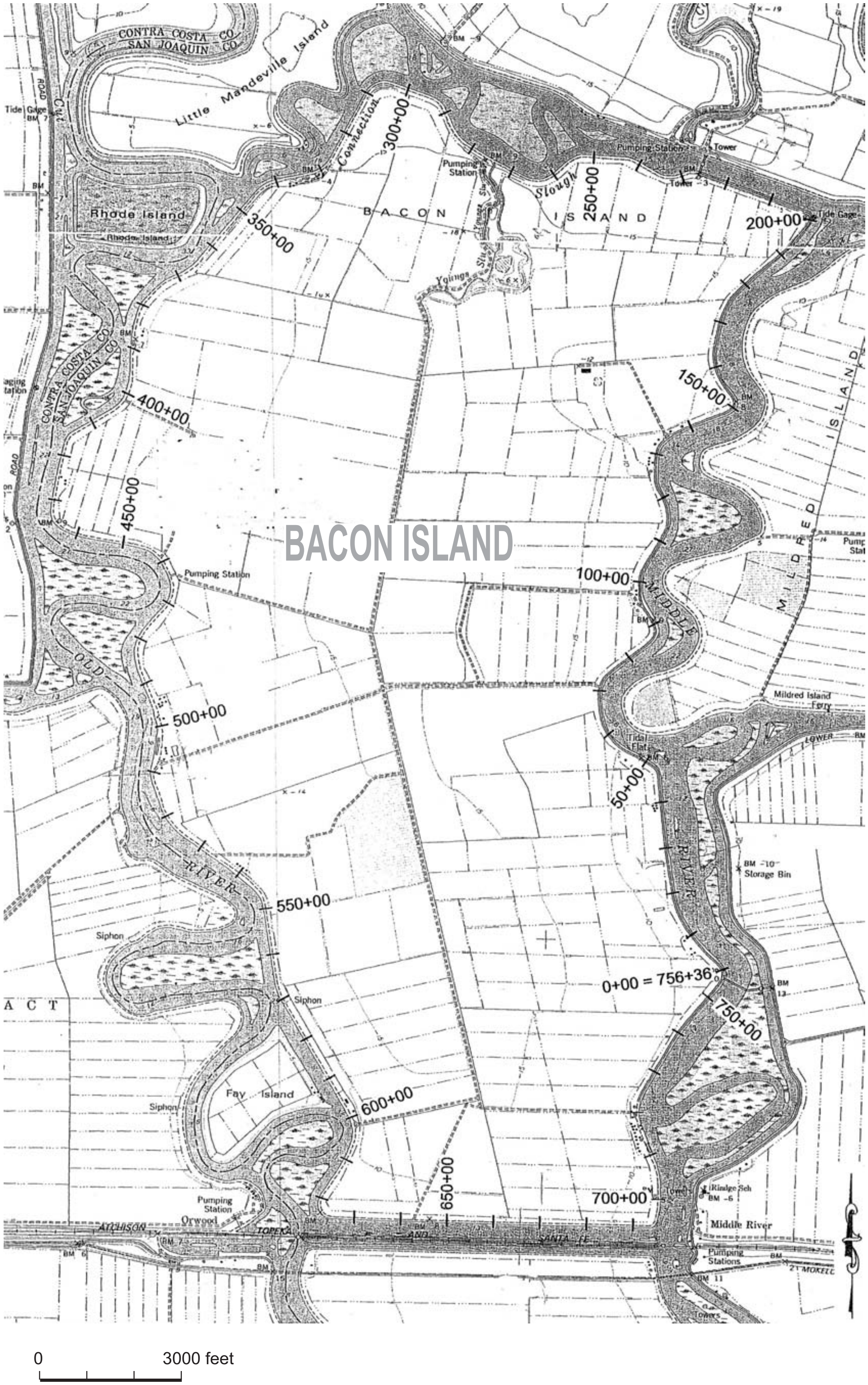
Project No. 26814104

WEBB Tract Reservoir

EMBANKMENT STATIONS AT WEBB TRACT RESERVOIR

Figure 3-1

26814104.01300/121102/gos



URS

Project No. 26814104

WEBB Tract Reservoir

EMBANKMENT STATIONS AT BACON ISLAND RESERVOIR

Figure 3-2

4.1 METHODOLOGY

Embankment breach analyses was made to estimate peak discharges from inward and outward breaches of Webb Tract and Bacon Island reservoirs and the resulting peak velocities and water surface elevations that could occur in the adjacent sloughs.

As discussed in Section 2.3.2, the slough widths were categorized as wide, medium, and narrow. In the analyses, the following assumptions were made in simulating hydraulic flow conditions at the breach opening and the opposite levee facing the breach:

1. Breach width of 400 feet was assumed based on previous dam breach studies for Webb Tract Reservoir (URS, 2000).
2. Time to breach was assumed to be 1.0 hour (MacDonald and Langridge-Monopolis, 1984).
3. The broad crested weir formula was used to calculate discharges through the breach opening under partially submerged conditions (Chow, 1959).
4. Under submerged conditions, the Bernoulli equation was used to calculate peak discharges across the breach opening accounting for head losses due to the sudden contraction and expansion of the flow through the breach.
5. The reservoir head during an outward breach considered the reduction in reservoir volume during the breach development time.
6. Breach was assumed to form in a straight reach of slough and develop perpendicular to the slough.

A two-dimensional hydrodynamic model (RMA-2) was used to determine the impacts of an outward reservoir breach of the embankment. The outcome from the analysis includes maximum flow velocities and maximum water surface elevations along the adjacent island levees.

For an inward reservoir breach, the higher peak discharges produce critical flows at the breach section. The RMA-2 model is not capable of simulating the critical flow regime. Therefore, normal flow conditions have been assumed to estimate flow velocities in the channel. The velocities near the adjacent islands are greatest on either side of the breach. As flow in the channel turns to pass through breach, velocities at the adjacent island embankment are reduced, approaching zero.

Table 4-1 provides the hydraulic head differential across the reservoir embankments (estimated based on data provided in Tables 2-1 and 2-2) and peak discharges used in the breach analysis.

Table 4-1
Hydraulic Head Differential and Peak Discharge

Breach Type	WSEL in Reservoir Island (feet)	WSEL in Slough (feet)	Head Differential (feet)	Peak Discharge (cfs)
Outward	+4.0	-1.0	5.0	95,000
Outward	+4.0	0.0	4.0	88,000
Outward	+4.0	+1.5	2.5	73,000
Inward	-8.0	+7.0	15.0	157,000
Inward	-8.0	+3.5	11.5	128,000

4.2 RESULTS

Model results show that during an outward breach, the water surface directly across from the breach rises significantly. Peak velocities are observed on either side of the breach near the banks of the adjacent island levees. As would be expected, velocities are relatively small on either side of the breach adjacent to the reservoir island embankment due to the formation of eddies.

During an inward breach of the reservoir, a similar flow pattern results, but the flow direction is reversed. Assumed slough levels that were analyzed were the 100-year flood stage (+7.0 feet) and the average-high tide (+3.5 feet). Both scenarios assumed that the reservoir is empty (at elevation –8.0 feet). This condition exists for about five months per year during the periods of emptying and filling of the reservoir (URS, 2001).

Peak velocities and water surface elevations estimated for narrow, medium, and wide slough sections are summarized in Tables 4-2 and 4-3, respectively. Peak velocities and water surface elevations presented are those observed near the adjacent island levee. Greater velocities are observed near the reservoir island breach. Further model results, including velocity distributions and water surface elevations estimated for the three slough sections, are presented in Appendix C.

Table 4-2
Estimated Peak Velocities for Typical Slough Sections

Breach Type	Head Differential[1] (feet)	Maximum Velocity (ft/sec)		
		Wide (3,000 feet)	Medium (1,000 feet)	Narrow (450 feet)
Outward	5.0	6.2	9.2	12.3
	4.0	5.4	8.0	10.7
	2.5	4.1	6.1	8.1
Inward	15.0	1.0	2.9	6.0
	11.5	1.0	2.8	5.8

(1) See Table 4-1.

Table 4-3
Estimated Water Surface Elevations for Typical Slough Sections

Breach Type	Head Differential[1] (feet)	Peak WSEL (feet)		
		Wide (3,000 feet)	Medium (1,000 feet)	Narrow (450 feet)
Outward	5.0	-0.1	0.5	1.7
	4.0	0.7	1.1	2.1
	2.5	1.9	2.1	2.7
Inward	15.0	7.0	7.0	7.0
	11.5	3.5	3.5	3.5

(1) See Table 4-1.

Figures 4-1 and 4-2 present the relationships between head differential and resulting peak velocity during a hypothetical outward and inward breach failure, respectively, for the three typical slough sections. These figures show that peak velocities are inversely proportional to the slough widths adjacent to the reservoir.

As discussed in Section 2.3.2, a velocity of range of 8 fps to 10 fps was selected as the threshold for failure of an adjacent levee. The results shown on Figure 4-1 indicate that the levees adjacent to narrow and medium slough sections would fail should the reservoir breach outward under the scenarios analyzed. Levee sections adjacent to wide slough sections would not fail during the outward breach scenario provided that they have slopes protected by riprap cover. Under an inward breach failure scenario shown on Figure 4-2, the adjacent island levees would not fail provided that they have slopes protected by riprap cover.

The average crest elevation of levees protecting adjacent islands is +8.0 feet (Section 2.3.2). Table 4-3 shows that there would be no adjacent island levee failures due to overtopping caused by an inward or outward breach of a reservoir island embankment.

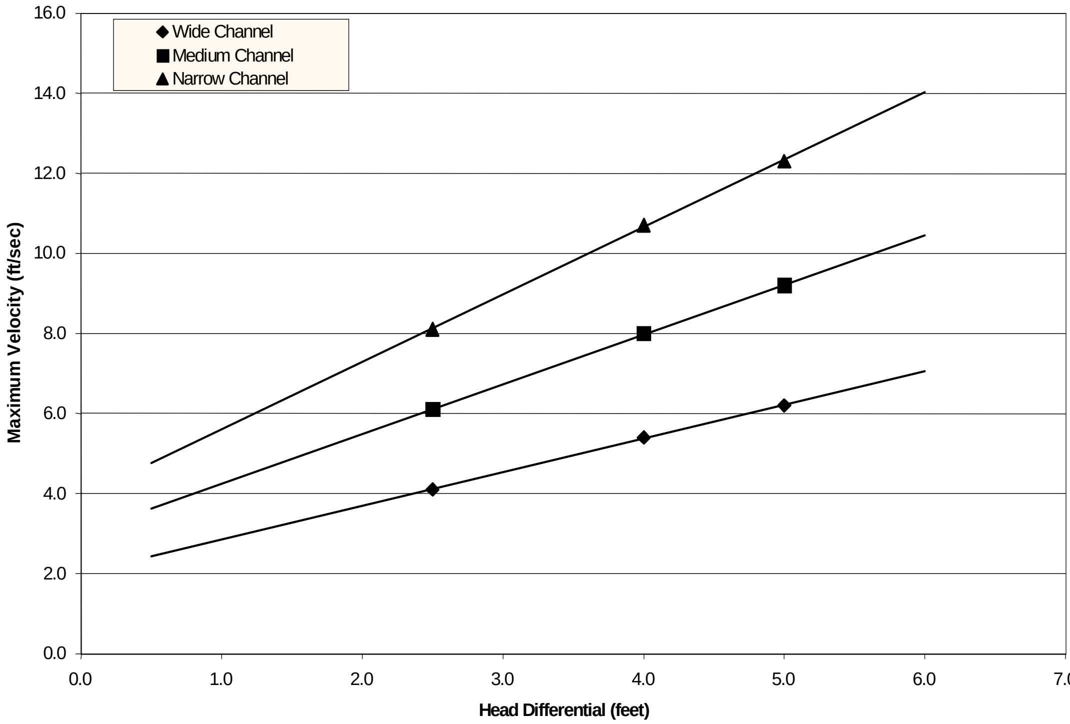


Figure 4-1: Results of Outward Embankment Breach Analysis

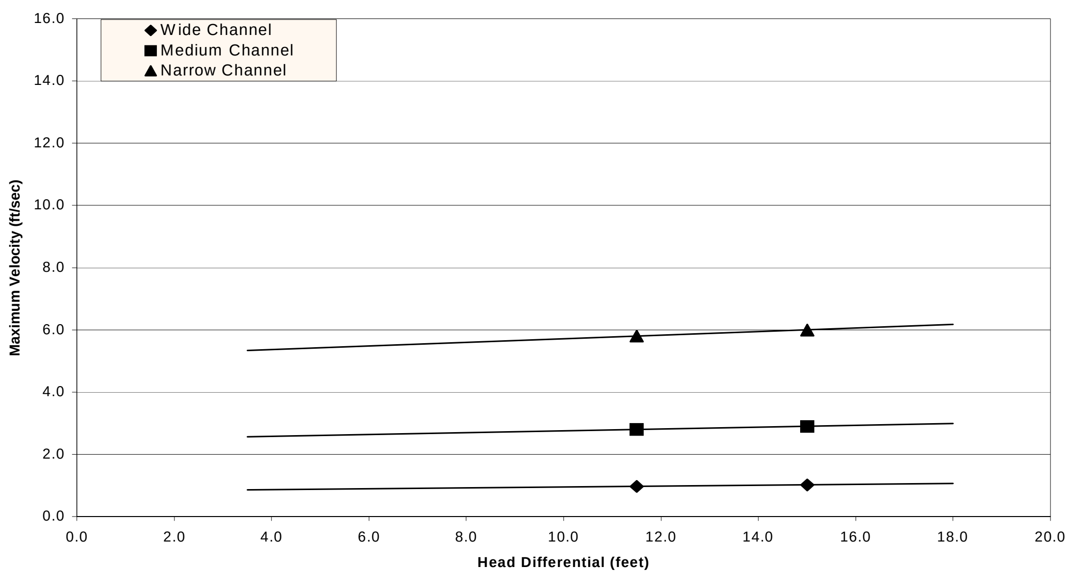


Figure 4-2: Results of Inward Embankment Breach Analysis

CALFED, May 2002, *In-Delta Storage Program, Draft Report on Engineering Investigations*, CALFED Bay-Delta Program, Department of Water Resources, Sacramento, California.

CALFED, September 1998, *Sacramento-San Joaquin Delta Levee Rehabilitation Study*, , Department of Water Resources, Sacramento, California

Chow, V.T. 1959. *Open-Channel Hydraulics*. McGraw-Hill Book Co. New York.

Harding Lawson Associates, September 1988. *Range of Particle Size Analysis*, Delta Wetlands, Sacramento River Delta, California.

MacDonald, Thomas C. and J. Langridge-Monopolis,1984. *Breaching Characteristics of Dam Failures*. Journal of Hydraulic Engineering. ASCE.

Neill, 1973, *Guide to Bridge Hydraulics*, Prepared by Project Committee on Bridge Hydraulics, Road and Transportation Association of Canada.

URS, November 2001, *In-Delta Storage Program Risk Analysis*, Prepared for US Bureau of Reclamation, Mid-Pacific Region and California Department of Water Resources.

USACE, February 1992, *Sacramento-San Joaquin Delta, California, Special Study, Hydrology*, US Army Corps of Engineers, Sacramento District.

USACE, 1984, *Shore Protection Manual*, Volume I and II, Forth Edition, Waterways Experiment Station, Vicksburg, Mississippi.

USACE, 1976, *Wave Runup and Wind Setup on Reservoir Embankment*, Engineering Technical Letter No. 1110-2-221, Washington, D.C.

USBR, 1992 Revised (1981), *Freeboard Criteria and Guidelines for Computing Freeboard Allowances for Storage Dams*, ACER Technical Memorandum No. 2, Denver, Colorado.